SPECIAL INVERTED-V-BRACED FRAMES WITH SUSPENDED ZIPPER STRUTS

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SUMMARY

This paper presents a simplified design procedure for an innovative bracing scheme labeled the suspended zipper frame. Conventional inverted-V-braced frames exhibit a design problem arising from the unbalanced vertical force generated by the lower story braces when one of them buckles. The unbalanced force must be carried by the floor beam, resulting in large beams and a relatively inefficient structural system. This adverse effect can be mitigated by adding zipper columns, or vertical members connecting the intersection points of the braces above the first floor. By introducing the concept of the suspension system, which consists primarily of an elastic hat truss at the top of the building, better behavior can be attained. The design procedure for suspended zipper frames and the performance of this system are illustrated with 3- and 9-story braced frames designed for similar loading as those used in the SAC model buildings. The proposed design strategy results in suspended zipper frames having more ductile behavior and higher strength than ordinary zipper frames. The suspended zipper frames also appear to reduce the tendency of chevron-braced frames to form soft stories and to improve seismic performance without having to use overly stiff beams. These are preliminary results and the procedure is still under development.

Keywords: suspended zipper frames, zipper frames, zipper columns, zipper struts, suspended zipper struts, chevron-braced frames, inverted-V-braced frames, V-braced frames, concentrically braced frames.

INTRODUCTION

Inverted-V-braced frames are one type of Concentrically Braced Frame (CBF), in which the centerlines of members form a vertical truss system to resist lateral forces. As more emphasis has been placed on increasing ductility and energy dissipation capability of all types of structures in modern codes, design provision for a new type of braced frame, labeled the Special Concentrically Braced Frame (SCBF), have been developed (Goel 1992, Bruneau et al. 1998). Within these provisions, the performance of Special Inverted-V-Braced Frames (SIVBF) was improved from that of ordinary Inverted-V-Braced Frames (IVBF) by limiting width/ thickness ratios, requiring closer spacing of stitches, and providing special design and detailing of end connections for the bracing members. However, SIVBFs still exhibit a typical braced frame design problem. Upon continued lateral displacement, the compression brace buckles and its axial capacity decreases while that of the tension brace continues to increase. This creates an unbalanced vertical force on the intersecting beam, resulting in a structural system that tends to concentrate inter-story drift in a single story, as shown in Fig. 1. In order to prevent undesirable deterioration of lateral strength of the frame, the provisions require that the beam shall possess adequate strength to resist this potentially significant post-buckling force redistribution in combination with appropriate gravity loads (AISC 2002). This results in very strong beams, much stronger than would be required for ordinary loads.

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Figure 1. Chevron Mechanism
The adverse effect of this unbalanced force can be mitigated by adding zipper columns, as proposed by Khatib et al. (1988) and shown in Fig. 2. The intent of SIVBFs with zipper columns is to tie all brace-to-beam intersection points together, and force all compression braces in a braced bay to buckle simultaneously. This results in a better distribution of energy dissipation over the height of the building. For instance, consider a SIVBF with zipper columns subjected to severe lateral loads. If the compression brace in the first story buckles while all other braces remain elastic, a vertical unbalanced force is then applied at the middle span of the first floor beam. The zipper columns mobilize the stiffness of all beams and remaining braces to resist this unbalance. The unbalanced force transmitted through the zipper columns increases the compression of the second story compression brace, eventually causing it to buckle. If the excitation is still forcing the structure in the same direction, the large unbalanced force will propagate up in the structure such that all compression braces are buckled. Near simultaneous brace buckling over the height of a building will result in a more uniform distribution of damage, a desirable goal. However, instability and collapse can occur once the full-height zipper mechanism forms, as shown in Fig. 2. This is due to the reduced lateral capacity of the frame after a full mechanism has formed (Tremblay and Tirca, 2003), and this drawback has limited the applicability of this system. Moreover, from a seismic design standpoint, a capacity design approach for zipper frames will require that assumptions be made both as to whether the zipper column or the tension braces should be allowed to yield and as to what the desirable deformation mechanism should be. These questions have not been answered decisively as yet.

Figure 2. Full-height zipper mechanism

Figure 3. Partial-height zipper mechanism

In this paper, the disadvantages of a full-height zipper mechanism will be overcome by introducing a suspension system, labeled "suspended zipper frame," as shown in Fig. 3. In a suspended zipper frame, the top story bracing members are designed to remain elastic when the all other compression braces have buckled and the zipper columns have yielded. Since the primary function of the suspended zipper struts is to sustain tension forces, and the suspended zipper struts support the beams at the midspan, the beams can be designed to be flexible. This results in significant savings in the amount of steel for the beams in SIVBFs with suspended zipper struts. Moreover, the force path is also so evident that a capacity design for all structural members is straightforward. In the following sections, the design methodology for the SIVBFs with suspended zipper struts is presented, and its use is illustrated with the design of the 3-and 9-story SAC model buildings. The performance of the structures is then assessed with the aid of nonlinear pushover and dynamic time history analyses. It should be emphasized that these are preliminary results and the procedure is still under development.

**PROPOSED DESIGN METHODOLOGY**

**Design philosophy**

As mentioned previously, the full-height zipper mechanism is potentially unstable once the full-height zipper mechanism forms. To overcome this weakness, a design procedure based on preventing buckling of the top-story braces is advocated here. This procedure suppresses the formation of a complete collapse mechanism and the formation of plastic hinges in the beams. As the lateral loads increase, the compression braces will buckle, the tension braces will yield, and finally the zipper strut will yield to form the mechanism shown in Fig. 3.
Design procedure

The design of Inverted-V-Braced Frames with suspended zipper struts can be accomplished by following a two-phase design procedure. In phase I (strength design), the frame is sized to resist the actions which result from the gravity and lateral loads applied to a conventional inverted-V braced frame (i.e., the zipper columns are not present and the structure is similar to that shown in Fig. 1). This phase fixes the size of the braces in all floors except the top story.

In phase II (capacity design), the zipper struts are added and other structural elements are redesigned except for the braces below the top-story level. The zipper column is designed to resist the vertical unbalanced forces generated by the braces below the floor under consideration, assuming \( P_y \) (not \( R_yP_y \)) for the braces in tension and 0.3 times \( \phi_cP_n \) for the braces in compression. The shear capacity of the beams is ignored. The decision to use a tension brace force of only \( P_y \) in the capacity calculations is intended to prevent excessive deformations in the tension brace and to force yielding in the zipper strut soon after tension yielding in the brace. The almost simultaneous yielding of the tension brace and zipper column prevents the concentration of drift in a single story.

The top-story braces need to resist both the vertical unbalanced forces and the top-floor level equivalent lateral earthquake force. A factor of 1.7 times the top-floor level equivalent earthquake force is proposed because the top-story braces need to be elastic throughout the load history. This 1.7 value needs to be verified further by considering the distribution of the system overstrength after the buckling of the compression braces.

In code language format, the design procedure can be summarized as follows:

1. Phase I (Strength Design):
   The frame shall be designed to resist the effects of earthquake and vertical loadings from the load combinations stipulated by the Applicable Building Code without the aid of the zipper columns.

2. Phase II (Capacity Design):
   The frame designed in Phase I shall be modified as follows:
   
   (1) Zipper columns
   Zipper columns shall be added and be designed to resist the vertical unbalanced forces generated by the braces located at the level below using \( P_y \) for the braces in tension and 0.3 times \( \phi_cP_n \) for the braces in compression.

   (2) Top-story braces
   Top-story braces shall be designed to resist elastically the vertical unbalanced forces collected by the zipper columns below the top story as well as the 1.7 times the top-floor equivalent earthquake force.

   (3) Column Strength
   The required axial compressive and tensile strength shall be determined using the maximum load transferred to the column considering the capacities of the adjacent braces in combination with the induced forces from the 1.7 times the top-floor equivalent earthquake force.

   (4) Beams
   Beams shall be considered as beam-column members stipulated at the Chapter H in the AISC LRFD Specification. The required strength shall be determined using the maximum load transferred to the beam considering the capacities of the adjacent braces.

EXAMPLES

3-story SAC model building

A special inverted-V-braced bay with suspended zipper struts, as shown in Fig. 4, was designed according to the proposed strategy. This structure corresponds to the lateral-load resisting portion of the three-story SAC model building. It was assumed that the beam-to-column connections as well as brace-to-beam and zipper column-to-beam connections are pinned. Unfactored uniformly distributed roof dead loads of 1.25 kips/ft, floor dead loads of 1.44 kips/ft, and live loads of 0.3 kips/ft are applied to the beams. The site is classified as site class D and its mapped spectral response acceleration at short periods, \( S_s \), and at 1 second period, \( S_1 \), are 1.5g and 0.6g.
respectively. The seismic weight for this braced bay is 1823.9 kips, which is a quarter of the entire building seismic weight. The seismic loads were calculated based on the value of \( R \) equal to 6, the value of \( I_c \) equal to 1.5, and the provisions of IBC 2000. Accordingly, the seismic base shear was calculated to be 456 kips, with the floor loads being 217.5 kips, 159.0 kips, and 79.5 kips from the third to first floor levels, respectively. The member sizes are listed on Table 1.

<table>
<thead>
<tr>
<th>Story</th>
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<th>Columns</th>
<th>Beams</th>
<th>Zipper</th>
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<tbody>
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A pushover analysis was performed using the OpenSEES program to estimate the maximum load and deformation capacities, as shown in Fig. 5. The brace model used is shown in Fig. 6. In stage 1, the structure is linearly elastic and the maximum base shear force is about 600 kips. This is significantly larger than the design base shear, which corresponded to 456 kips. When the first story compression brace buckles, the structure enters stage 2. When the second story compression brace buckles the structure enters stage 3 and the maximum base shear is reached. At this stage the maximum base shear is 715 kips or about 1.44 times the design base shear. Once the first story tension brace yields, the structure goes into stage 4, which is characterized by a slow loss of strength as the first story tension brace continues to yield. When the first floor compression brace reaches its constant post-buckling value and its stiffness becomes a small positive value, the structure enters stage 5. In the stage 6, the structural strength decreases again due to the yielding of the second story tension brace. Once the second story compression brace reaches the minimum post-buckling strength, the structure enters stage 7 and its strength increase slightly until the floor beams begin to collapse in stage 8. This phenomenon is consistent with the mechanism shown in Fig 3. The results of this pushover analysis indicate that the design procedure results in a structure that is ductile and not susceptible to large losses of lateral strength and stiffness as conventional braced frames are.

![Figure 4. Elevation of the zipper braced bay](image)

![Figure 5. Pushover curve for the 3-story SAC model building.](image)
In order to examine the dynamic performance of this building model, nonlinear dynamic analyses were performed using the OpenSEES computer program. For the simulation of bracing members, the hysteretic uniaxialMaterial option which sets up the relationship between the member’s axial forces and deformation was selected (See Fig. 7). For this member, pinching factors (pinchX 0.5 and pinchY 0.3) were assumed to model the amount of pinching of the deformation and force, respectively, during reloading. A damage factor of 0.02 for damage due to ductility was also selected. Beams and columns were modeled using beam-column elements with the fiber section function that can track the yielding status of the prescribed fibers in the sectional flanges. P-δ effects were considered, and 5% Rayleigh damping was specified in the first and third modes of vibration. The computed periods of the structure in its first three vibration modes were 0.478s, 0.161s, and 0.100s.

![Figure 6. Brace model for pushover analyses.](image)

![Figure 7. Brace model for time history analyses.](image)

Figure 8 shows the sequence of brace yielding, zipper strut buckling, and beams and/or columns full-or partial-section yielding when the 3-story SAC model building is subjected to the Kobe earthquake (LA21). The solid rectangles represent brace buckling, while the solid circles represent braces, zipper struts, columns or beams yielding. Beginning about 7.56 sec., when the largest demand occurred, the structure moved left, and all left compression braces below the top level buckled. The first- and second-story tension braces then yielded, followed immediately by the yielding of the upper zipper column. Compression yielding and tension yielding occurred on the top flange of the right first-floor beam and on the bottom flange of the left second-floor beam, respectively. At 8.93 sec, the top floor had the maximum drift, 13.71 in, whose corresponding structural deflection shape is shown in Fig. 9. At this point, interstory drifts corresponded to 0.20%, 2.86% and 5.73% for the third through the first floor respectively (fig. 9). Although the deformation is concentrated in the first floor, as expected, the distribution of interstory drifts is reasonable.
9-story SAC model building

Another design example is for an inverted-V-braced bay with zipper columns corresponding to the 9-story SAC model building was also carried out. An elevation is shown in Fig. 10. The seismic weight for this braced bay is 5814 kips which is a quarter of the entire building seismic weight. The designed seismic base shear is 991 kips distributed over the model height based on the IBC 2000 equivalent lateral force procedure. The member sizes, as listed on Table 2, were determined using the previously proposed design methodology.

Table 2. Member sizes for the 9-story SAC model building

<table>
<thead>
<tr>
<th>Story</th>
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<th>Columns</th>
<th>Beams</th>
<th>Zipper</th>
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<tr>
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<td>W14x257</td>
<td>W14x90</td>
<td>W14x159</td>
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<tr>
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<td>W14x257</td>
<td>W14x90</td>
<td>W14x132</td>
</tr>
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</tr>
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</table>

The pushover curve for this structure is shown in Fig. 12. The maximum base shear is about 1200 kips, which is 1.21 times larger than the design seismic base shear. In this case, however, there is a large drop in strength (about 370 kips) after the peak base shear. This results from the simultaneous buckling of all the compression braces except for the top ones. Once the majority of the compression braces reach their minimum post-buckling strength, the structural strength will increase gradually until the tension braces start to yield. Finally, a failure mechanism forms as one beam becomes unstable. As seen on Fig. 12, the maximum top floor displacement is 90 in., which indicates a 6% drift capacity.

In the nonlinear dynamic analyses, the computed periods of the structure in its first three vibration modes were 1.42s, 0.46s, and 0.23s, respectively. A 5% Rayleigh damping was specified in the first and ninth modes of vibration, which ensured that all modal damping ratios did not exceed 5%. The dynamic behavior of this model was examined by performing nonlinear time history analyses with all earthquakes from the SAC suite of ground motions (LA01–LA40). The result of the analysis for the 1994 Northridge earthquake with a PGA of 1.33g (LA28) is shown in Fig. 10. Interestingly, during the peak motion for this earthquake, the structure first moved to the left and all left compression braces but the top one buckled, initiating at the base and propagating towards the top floor. Meanwhile, some of the right tension braces yielded, and the second- and third-story zipper struts yielded too. When the structure moved to the right, all right compression braces buckled, but initiating at the top floor. Fig. 11 illustrates the deflected shape for the maximum top floor displacement of 22.28 in. at time equal 3.89 sec under the LA28 earthquake.
The mean plus one standard deviation, mean, and maximum values of the peak inter-story drift ratios for the ground motion ensembles with 10% probability of exceedence in 50 years (earthquakes LA01–LA20), and with 2% probability of exceedence in 50 years (earthquakes LA21–LA40), are presented in Fig. 13 and Fig. 14, respectively. The former figure shows a fairly uniform mean distribution of inter-story drifts over the height. The mean plus one standard deviation shows a similar pattern, while the maximum distribution shows a concentration of deformations in the second floor. For the particular case of the LA38 earthquake and the 9-story structure, the maximum inter-story drift ratios are 6.32%, 8.08%, 7.98%, 6.32%, 4.35%, 3.22%, from the first floor up to the sixth floor, respectively. At this time instant, all the left compression braces buckle except for

Figure 10. Sequence of buckling and yielding under LA28.

Figure 11. Deflected shape at 3.89 sec under LA28.

Figure 12. Pushover curve for the 9-story SAC model building.
the top one, and the first- through seventh-story tension braces as well as the first- through eighth-story zipper struts yield due to the significant 1.2g ground acceleration. Since the yielding in the tension braces starts at the base toward the top floor, the lower-story inter-story drifts are larger than those on the upper stories.

Figure 13. 10% exceedence in 50 years (LA01–LA20)  
Figure 14. 2% exceedence in 50 years (LA21–LA40)

CONCLUSIONS

The proposed design strategy results in suspended zipper frames having more ductile behavior and higher strength than ordinary zipper frames. The suspended zipper frames appear to reduce the tendency of chevron-braced frames to form soft stories and to improve seismic performance without having to use overly stiff beams. The main disadvantage of the proposed configuration is that as the number of stories increase, the magnitude of the vertical unbalanced forces transmitted up to the top-story braces becomes very large, making the design of the top story braces difficult. Therefore, a limitation on the height for this system will probably be necessary.

REFERENCES